

Characteristics of Liquefaction-Induced Damage in the 2011 Great East Japan Earthquake

K. Ishihara

Research and Development Initiative, Chuo University, Tokyo, Japan. E-mail: kenji-ishihara@e-mail.jp

K. Araki

Design Department, Chemical Grouting Co., Ltd, Tokyo, Japan. E-mail: k-araki@chemicalgrout.co.jp

B. Bradley

Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand. E-mail: brendon.bradley@canterbury.ac.nz

Keywords: Large earthquake, Liquefaction, Damage, Lateral flow, Settlements, Soil Characteristics.

ABSTRACT: Features of the 2011 earthquake in Japan are characterized by predominance of the ground failure due to liquefaction and scour of the ground caused by Tsunami. Unprecedented long duration of the shaking and extensive occurrence of liquefaction in Tokyo Bay area 300~400km distant from the epicentral area are also the characteristic features of the earthquake. Many specially designed tsunami seawalls were destroyed by the raid of the giant tsunami as high as 15~20m in the coastal region of the northern part of Japan mainland. The results of preliminary investigations and simple analysis on the liquefaction-induced damage are introduced herein, together with lessons learned from the destruction.

1. INTRODUCTION

An extremely large earthquake rocked the widespread area in the northern part of Japan Mainland at 14:46:33 (JST) on March 11, 2011. The magnitude of the quake was $M=9.0$, an event accompanied with an unprecedentedly large amount of energy release at a centroid depth of 24km where the Pacific tectonic plate subsides into the Okhotsk plate underneath the Japan Mainland. This quake was the largest ever recorded during the last 150 years since the start of seismic observation in Japan. Subsequently, the two big-scale ruptures of faults occurred, one at 15:09 with $M=7.4$ in the north and the other at 15:15 with $M=7.7$ in the south on the same day as displayed in the map of Figure 1.

The fault zone covered a wide area about 500km long and 200km wide. It is noted that the biggest first quake with whopping $M=9.0$ occurred at an epicenter under sea 175km off the coast of Sendai, followed by the second largest event at 15:08 with epicentre about 150km east of northern coast. The fault ruptures moved southward, generating the third largest aftershock at 15:15 at a southern site 30km off-shore from the coast of Ibaragi prefecture. Apart from the raid of the series of tsunami, strong shaking was felt and recorded at a number of sites over a widespread area from the northern prefectures, Iwate, Miyagi and Ibaragi, down to the southern region such as Chiba and Tokyo metropolis. Extensive liquefaction developed in the

area of Tokyo Bay as well as over the flat lowlands surrounding lower reaches of the Tone River north of Tokyo. The features of liquefaction and associated damage will be described in the following pages of this paper.

2. GROUND MOTION CHARACTERISTICS IN THE EPICENTRAL AREA

Shown in Figure 2 are time histories of accelerations monitored at the K-Net station in Sendai, the area most severely shaken. It is noted that the peak acceleration monitored in the first $M=9.0$ event was 1000gal in east-west direction (E-W component) and the duration of the main shaking was as long as 180 second. In the second quake that occurred 15 minutes later, the peak acceleration was 900gal with the duration of about 120 second. The trajectory of recorded accelerations projected on the plane of East-West and North-South axes is shown in Figure 3 where it is noted that the largest shock had occurred in the N-S direction with the acceleration as much as 2000 gal. The time histories of velocity obtained by integration of the recorded accelerations are shown in Figure 4 where it is noted that the maximum velocity in the first event was as much as 80cm/sec. and that in the second quake was 70cm/sec.

The response spectra of recorded accelerations and computed velocities are displayed in Figures 5 and 6,

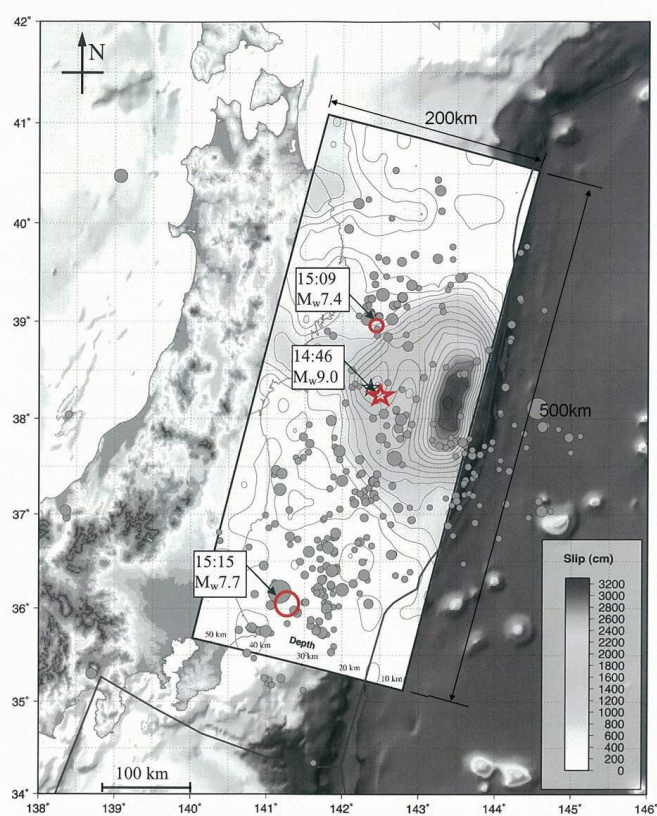


Figure 1 Epicentral zone of the 2011 Great East Japan Earthquake (Hayes et al. 2011)

respectively, where it can be seen that the peak response acceleration and velocity occurred at the period of 0.7~1.0 second. It is also to be noted that the velocity at a period of 1.5 second is fairly predominant.

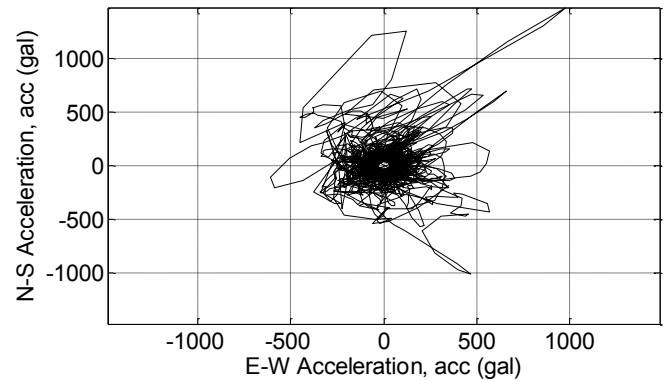


Figure 3 Trajectory of recorded motion in Sendai projected on the E-W and N-Z plane (values less than 50gal are removed).

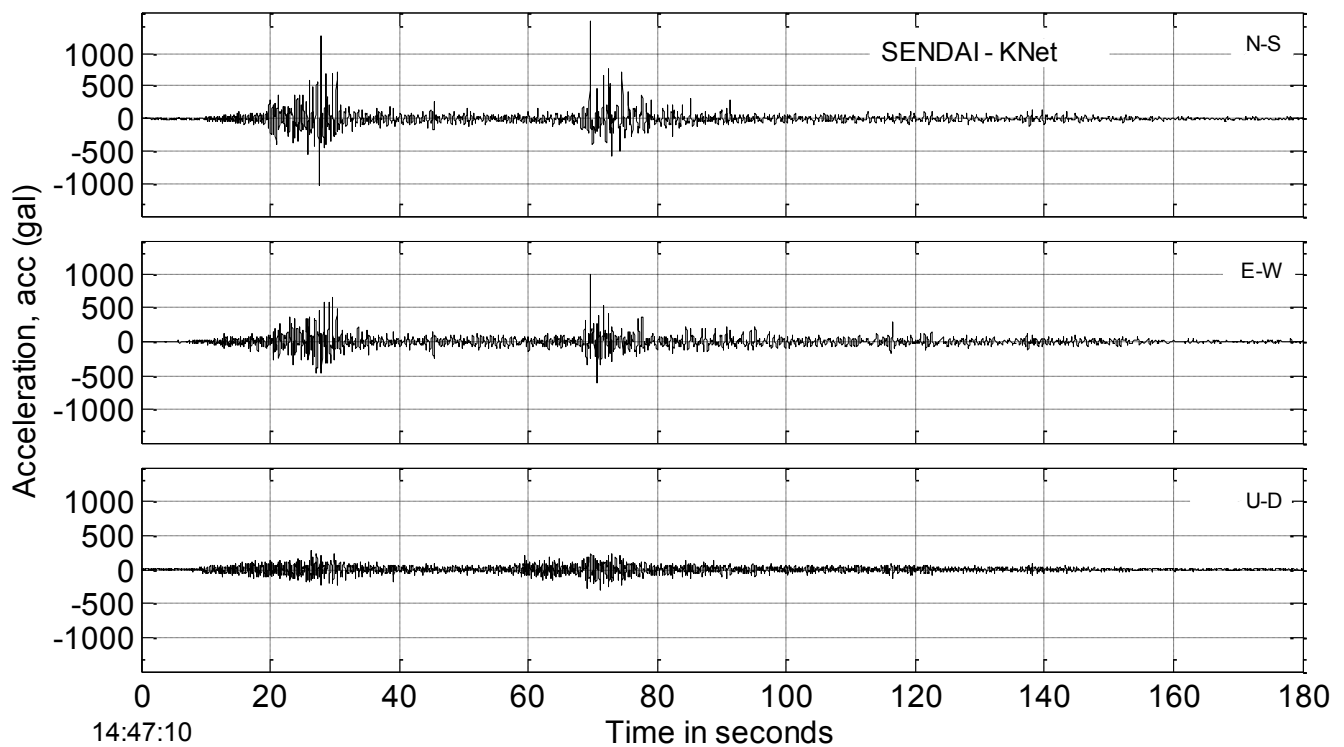


Figure 2 Recorded motions at the Sendai K-Net station

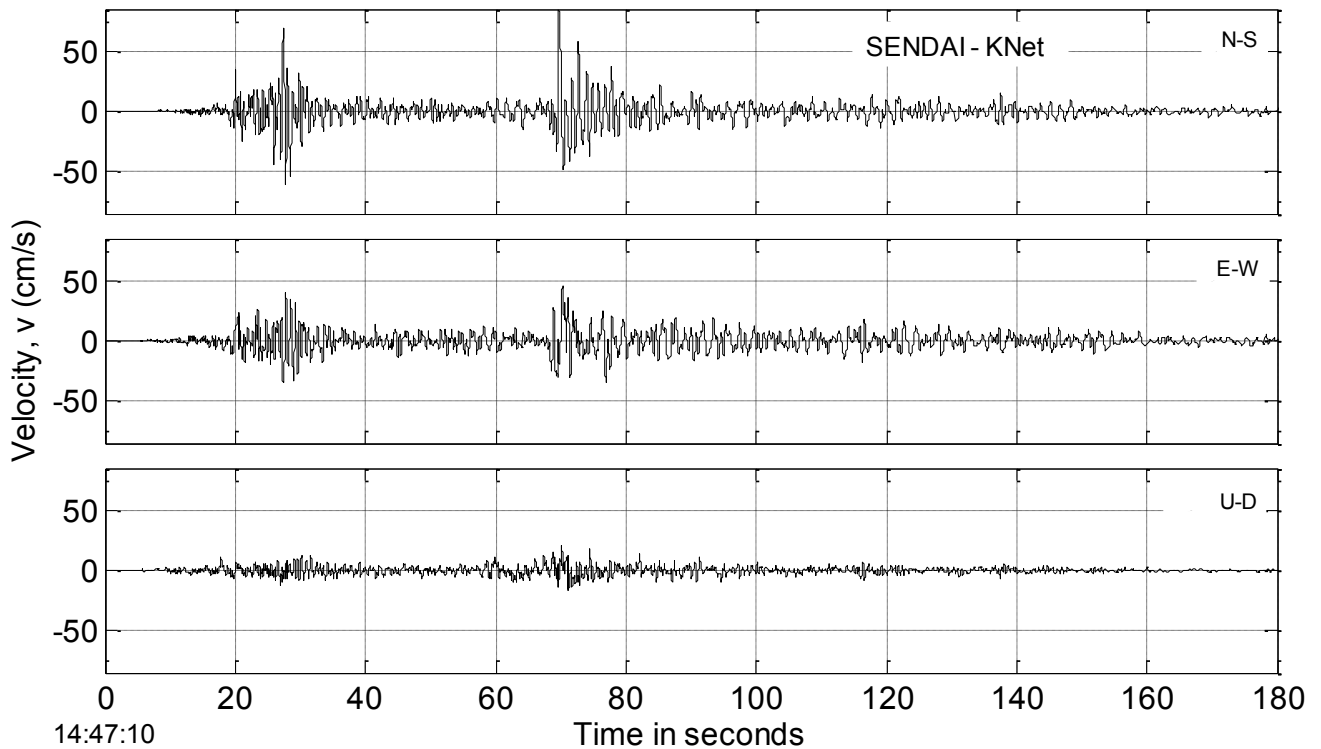


Figure 4 Time histories of velocities obtained from recorded accelerations at the Sendai K-Net station

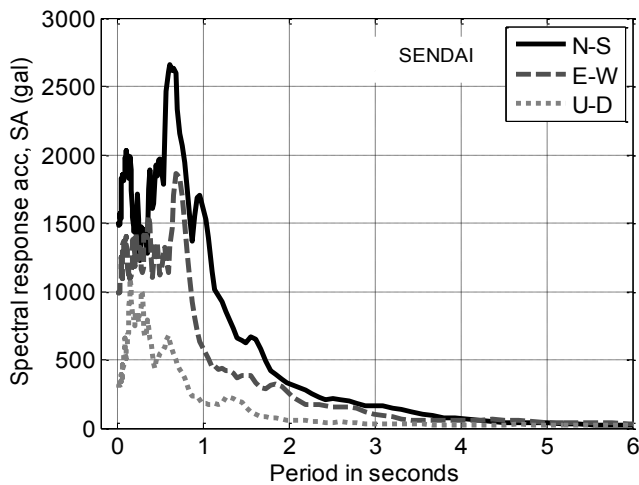


Fig. 5 Accelerations spectra at the Sendai K-Net station (for 5% of critical damping).

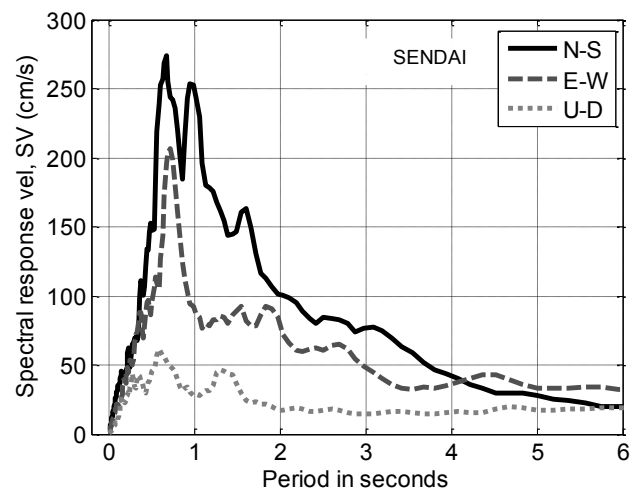


Figure 6 Verocity response spectra at the Sendai K-Net station (for 5% of critical damping)

3. GROUND MOTION CHARACTERISTICS IN TOKYO REGION

There are several K-Net and Kick-Net stations installed in the Kanto region where strong motion recorders were triggered. Locations of some of these stations are shown in Figure 13. Typical motions obtained at Urayasu K-Net station is demonstrated in Figure 7. It is noted here that the main shock by the largest quake

with $M_w=9.0$ was recorded at Urayasu 107 seconds after the start of the motion at Sendai. This difference is the time required for the tremor to travel through the distance of approximately 350km from Sendai to Tokyo area. Thus, the velocity of propagation of the first wave front is estimated to have been 3.1km/sec. It can be seen in Figure 7 that the peak horizontal acceleration (PGA) was 160g in E-W direction.

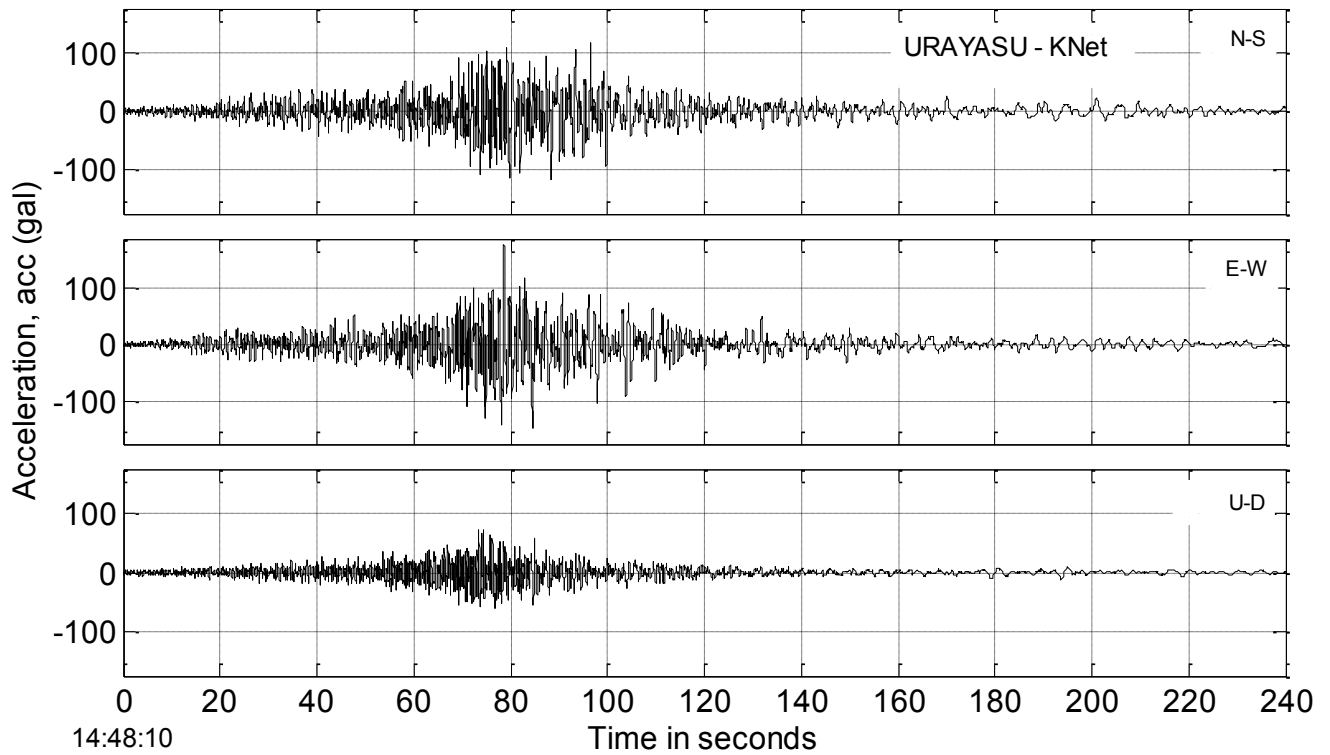


Figure 7 Recorded motions at the K-Net station at Urayasu.

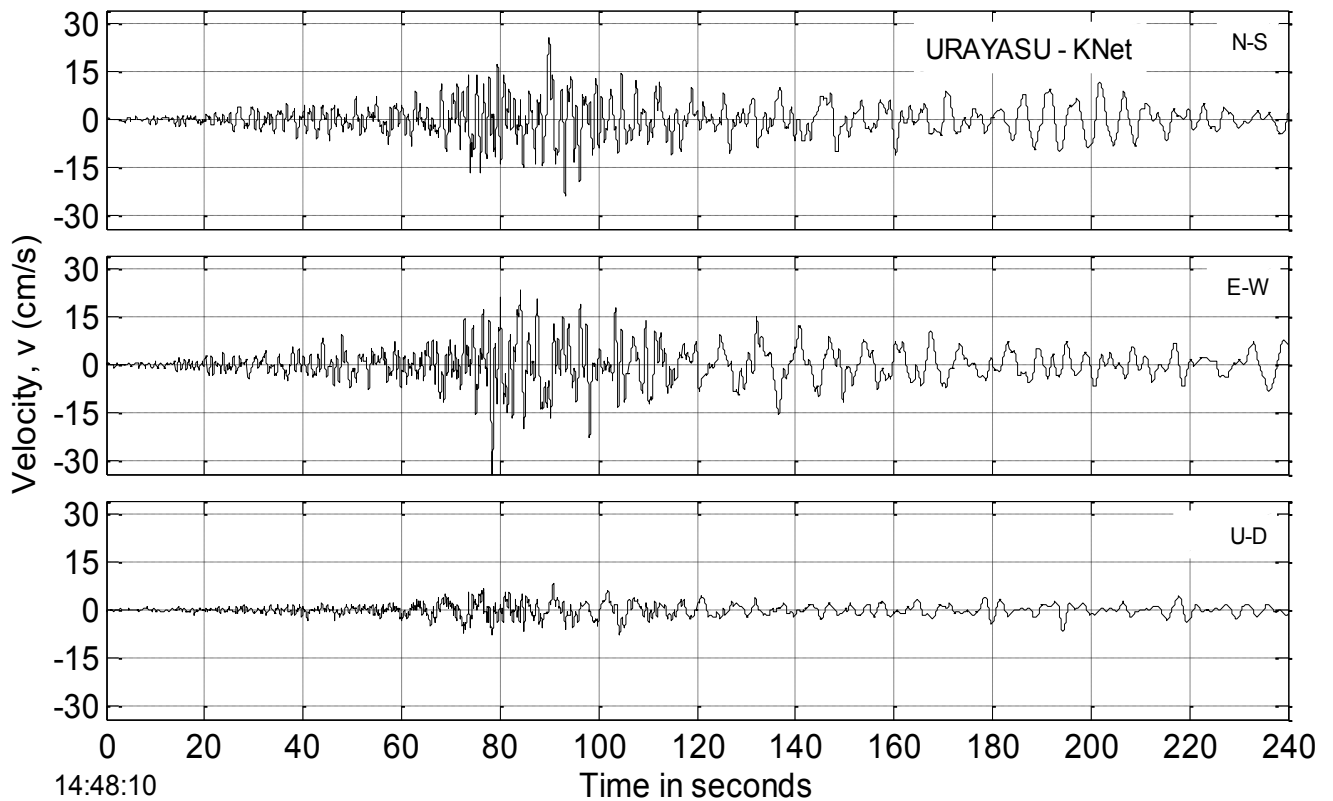


Figure 8 Time histories of velocities obtained from recorded accelerations at Urayasu

A majority of records obtained in Tokyo Bay area had a PGA in the range of 150 to 200 gals, and that recorded at Urayasu is considered as being representative of many others in this area. The time histories of computed velocities in Urayasu are shown in Figure 8 where it is noticed that the peak value is of the order of 30 cm/sec. The trajectory of the recorded acceleration at Urayasu is displayed in Figure 9 where it is noted that the predominant motion was in the East-West direction. It is to be noticed in Figure 7 that the main shaking in Tokyo lasted as long as 150 seconds which are considered as being larger, perhaps, than any others ever recorded in the world. It should be mentioned herein that such a long duration of motions has been the major cause of the liquefaction-related damage which were vastly disastrous as described below.

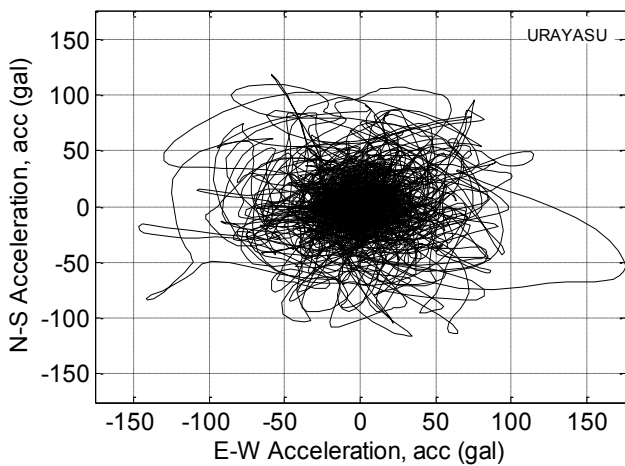


Figure 9 Trajectory of recorded motion at Urayasu K-Net station plotted on the E-W and N-S plane

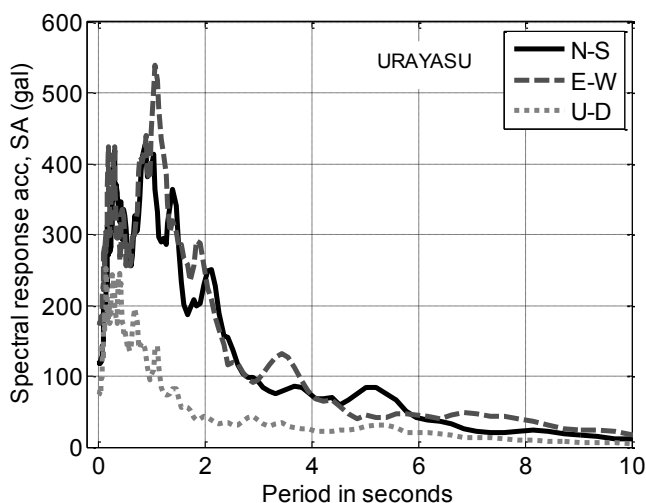


Figure 10 Response of accelerations recorded at the Urayasu K-Net station (for 5% of critical damping)

The response spectra of the recorded acceleration at the Urayasu K-Net station is displayed in Figure 10. It is noted that the peak spectral acceleration occurred at the period of about 1.0 seconds. The spectral velocity at Urayasu is demonstrated in Figure 11 where it may be seen that the large response occurred at the period of 1, 2, 3.5 and 5 seconds.

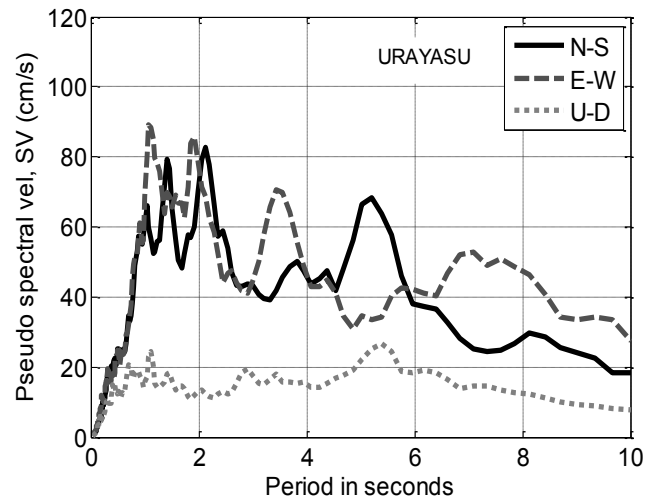


Figure 11 Response spectra of velocities at Urayasu (for 5% of critical damping)

4. CHARACTERISTIC FEATURES OF THE GROUND DAMAGE

4.1 The damage in the north Tohoku region

The eastern part of the north Tohoku region is generally mountainous with steep rocky slopes dipping sharply into the sea. There are dozens of small canyons and inlets formed by river channels along the coast from Hachinohe in the north down to Sendai in the south. This coastal geomorphology known as the “rias” formation is characterized by the presence of steeply incised valleys which are filled with deposits of Pleistocene era. In such topography along the coast, the height of tsunami is amplified significantly in the course of its invasion into the inlet, resulting in the apocalyptically dreadful disaster. This has been indeed the case in the past with the tsunami-generated devastation in the north Tohoku region. There might have been liquefaction-associated damage by the seismic shaking, but because of significant erosion and scour of the subsurface soil deposits caused by repetition of inflow and retreat of the tsunami, traces of the ground damage due to liquefaction, if any, seem to have been washed away, and could not be identified visually in the investigation after the event.

4.2 Liquefaction in Kanto region in the south

Extensive liquefaction was induced in reclaimed and alluvial deposits along rivers and bay areas in the plain of Kanto region including Ibaragi, Chiba and Tokyo which are located as far as 300~400km south west of the epicenter of the main shock. The total land area in which signs of liquefaction were observed is purported to have been of the order of 70km². In terms of the long distance from the epicenter and also in terms of the large expanse net area, the liquefaction in the Kanto region by the 2011 earthquake was unprecedented and truly record-breaking. Shown in Figure 12 is the distribution of places where apparent signs of liquefaction were observed such as sand oozing, boiling, ground cracking and associated ground settlements. These may be classified as (i) those which developed in the reclaimed waterfront area along the Tokyo Bay, (ii) the liquefaction in the lakeshore district south of Kasumiga-ura in the lower reach of Tone River, and (iii) the spotwise occurrence of liquefaction at many places along the Tone River and its tributaries where landfills had been conducted in originally marshy flat lands. Somewhat detailed account on the features of the damage is given below, focusing on those in the area of Tokyo Bay.

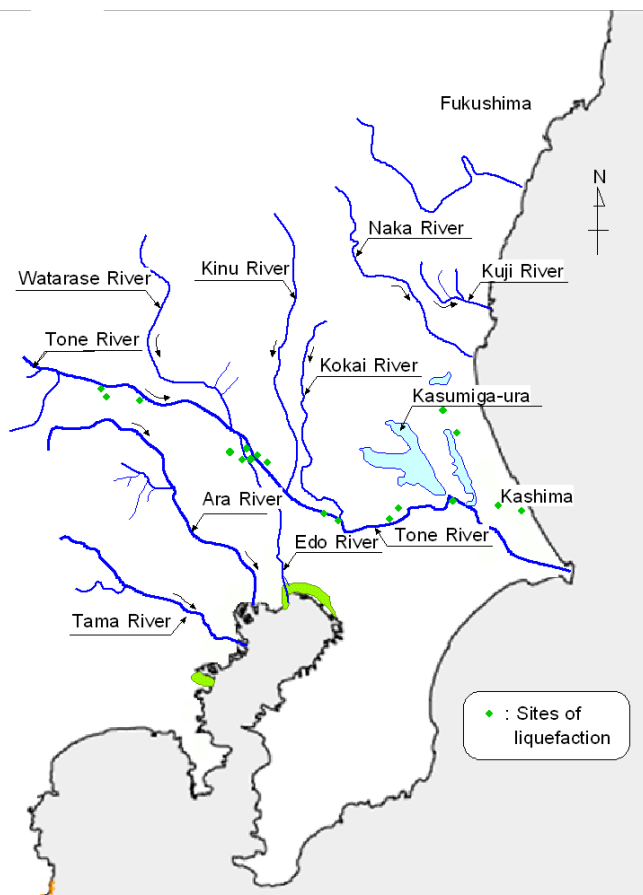


Figure 12 Liquefaction - affected area in Kanto Region (S. Yasuda, 2011)

4.3 Typical types of damage due to liquefaction in Kanto region

One of the features specific to the present event was the occurrence of liquefaction over the area farther than any location ever recorded. The areas affected in Kanto area do span from the water-rich flood plain in the Tone River in the north to the alluvial or reclaimed areas in Tokyo Bay in the south. The distance from the epicenter was in the range as far as 300~400km. This appears due to the largest magnitude of the earthquake $M=9.0$ ever encountered in the Japan archipelago since seismic observation started about 130 years ago. The distress to buildings and infrastructures caused by the liquefaction may be classified roughly into four types as described below.

4.3.1 Liquefaction in the level ground in Tokyo Bay area

There were a countless number of places where apparent signs of liquefaction were observed in flat ground. These observations included parks, play ground of schools, soccer fields, baseball fields and etc. Sand boiling and oozing occurred, spurting a considerable amount of sand or silt accompanied by ground settlements as much as 10~100cm all over the nearby area. One of the characteristic features was the fact that the sand spurting did occur preponderantly along long cracks or fissures which were opened in the level ground with a linear length of 2~10m. This fact appears to indicate that not only horizontally polarized shear wave but also surface waves or reflected waves might have been induced in this area. It is also to be mentioned that the huge amount of silt or sand spurting produced deposits as thick as 50cm on the ground surface. It is thus suspected that large voids of various sizes are left under the ground.

4.3.2 Liquefaction around buried lifelines

There are a number of complicatedly knit networks of pipelines such as sewage pipes, water supply pipes, and gas supply lines. These were generally embedded during various eras in near-surface deposits at depths 1~2m, by excavating ditches and backfilling by sandy soils. Many times, these lifelines were laid down in sand deposits but in some cases in clayey or silty deposits containing gravels. No matter whether they lie in sandy or clayey soils, the backfilling was made by sands without compacting them sufficiently. Thus, it has been quite common in the past earthquakes to observe uplifting of the manholes and sometimes pipes themselves particularly for sewage drainage lines. The same type of damage occurred widely in the reclaimed areas in Tokyo Bay.

4.3.3 Effects of liquefaction on buildings and houses

When buildings are small or flat, they are typically constructed on flat or continuous reinforced concrete footings which are placed at depths 0.5~1.0m directly on the underlying near-surface soil deposits. If the sandy soils underneath such structures had not been stabilized, liquefaction often developed readily causing deleterious effects on the foundations of houses involving settlements which were more or less accompanied by varying amount of tilt. Even in the case of a small tilt of the order of 0.005, houses became uninhabitable. The tilt of private residences accompanied by overall settlement of the order of 10~70cm was observed over a number of residential sections. Buildings higher than 3-stories, which are typically constructed on pile-supported foundations or on the flat foundations sitting on improved near-surface deposits remained intact and usable.

4.3.4 Flow slides in gentle slopes

Near the waterfront area and sloping ground as well where there is difference in elevation, lateral displacement or slides occurred at many places with horizontal displacement on the order of 0.5~3.0m. In a long mileage of coastal line along the Tokyo Bay, the waterfront is protected by different types of seawalls retained by mounds of stones or stacks of wave-breaking concrete blocks.

The areas of liquefaction are roughly displayed in Figure 13 where it is noted that its effects extended south to the city of Kanazawa-bunko and also that the low-land areas west of the lower reach of the Edo River in Tokyo suffered the damage due to liquefaction. Several cities in Chiba Prefecture east of Urayasu were affected more seriously by the liquefaction. These cities are shown in Figure 14.



Figure 13 Liquefaction-affected areas in the Tokyo Bay

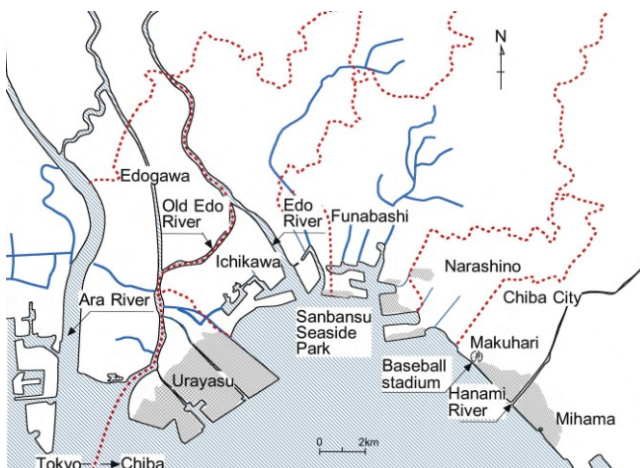


Figure 14 Liquefaction-affected areas in Chiba along the Tokyo Bay

5. LIQUEFACTION-AFFECTED TOKYO BAY AREA

5.1 Damage in Urayasu City

Amongst the cities in Chiba Prefecture, Urayasu was the one most seriously damaged by liquefaction. The major towns in Urayasu are indicated in Figure 15 where apparent signs of liquefaction were observed at the time of the first strongest shaking on March 11, 2011. Most of the areas affected were those reclaimed over the period of 1960~1980 and developed to provide landspace for residential houses, parks, schools and warehouses for industries. There was a sharp contrast between the areas of liquefaction or no liquefaction which are divided by the old shore-line before the reclamation started each in 1960's. This boundary line is indicated on the map in Figure 15.

Most severely affected areas in Urayasu were districts where land expansion was undertaken towards the shallow sea waters starting from early in 1960's. The progress of land reclamation is indicated in Figure 16 where it can be seen that the filling in the reclaimed area had been finished by the end of 1970's. The landfiling was conducted by dredging shallow-depth seabed near the shores. The sandy soils were transported hydraulically by pipes and sedimented under water. The photos in Figure 17 show the reclamation works then underway. After filling hydraulically, dry sand was transported onland and spread by bulldozers to a thickness of 1~2m. No means was taken further to compact the soils. Soil improvements were left to the hands of users who purchased the land. Generally speaking, the places where large buildings and infrastructures were to be constructed, soils were stabilized by means of vibro-compaction or other techniques. Individual spots where evidences of liquefaction were observed are indicated in

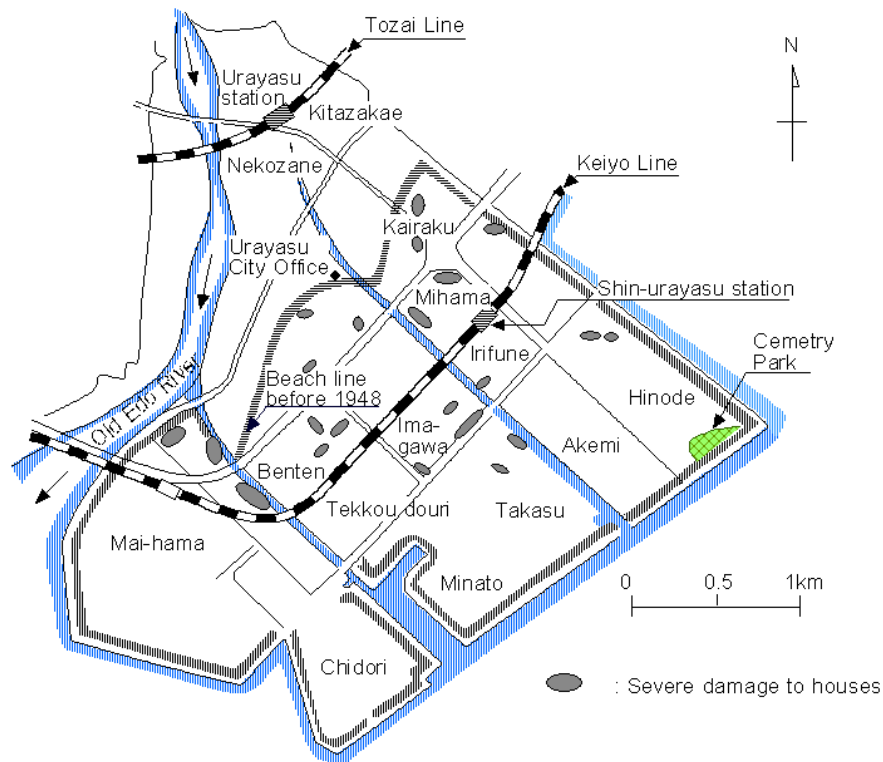


Figure 15 Places of the serious damage to private houses in Urayasu

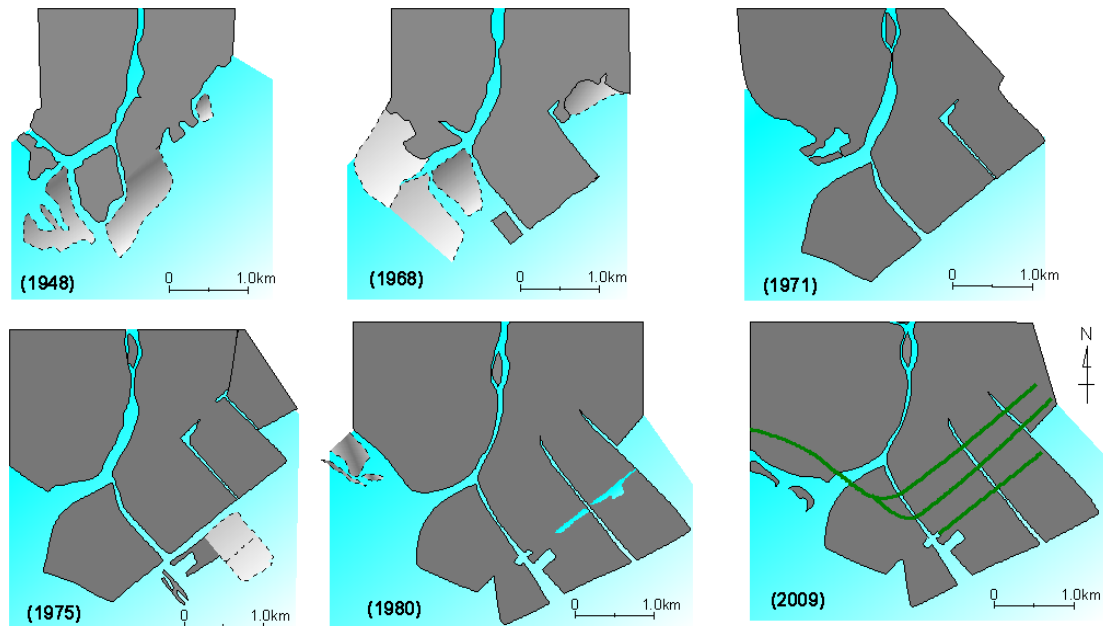


Figure 16 Progress of reclamation work in Urayasu (Urayasu city office Homepage)

Figure 15 where it is noticed that the area north of the old beach line as of 1945 remained intact, not affected by liquefaction, in spite of the lower elevation of the ground surface as compared to that in the near-shore reclaimed area. The soil profile at Urayasu consist generally of loose sand layers of reclaimed and alluvial

origin underlain by a thick soft clay-silty layer to a depth of 40~70m. A typical soil profile at a site at Irifune 3-chome is shown in Figure 18. It can be seen that a soft clayey silt deposit of alluvial origin exists to a depth of 45m at this site overlaid by alternate layers of silty and sandy soils of alluvial era.

The loose sandy layer near the surface to a depth of 5m having a SPT N-value of 3~10 is the one placed newly by the reclamation. A series of boring data obtained along the alignment D-D' in Figure 20 are arranged in a form of a side view as shown in Figure 19. It is of interest for notice that although the SPT N-value of 5-15 for the alluvial sand is slightly larger as compared to N=5~10 for reclaimed sand, the new sand deposit was more vulnerable to liquefaction, as evidenced by the clear manifestation of liquefaction in the reclaimed area in contrast to no liquefaction in the old area in the north. This fact seems to indicate that effects of solidification of fines due to aging contributed for strengthening of sandy soils although it is not clearly reflected in the penetration resistance of the SPT. Upon compilation of many other boring data, the bottom of the alluvium was determined and shown in Figure 20 as contour lines of buried valleys.



(a) Distant view



(b) Pipes of hydraulic transmission

Figure 17 Photos showing the reclamation work by the hydraulic method (offered by Urayasu city office)

Several of photos showing the ground devastation by liquefaction are shown in Figure 21. Figure 21(a) shows tilting of trees and electric cable-supporting blocks at a house in Mihama residential section. Shown in Figure 21(b) is uplifting of a sewage manhole about 1.5m above the ground. The features of the ground damage in general are shown in Figure 21(c). It should be noted that the ejected sand was mostly laden with silt and clay, making it easy to flow and spread over the ground surface. Figure 22(a) shows a photo indicating settlement and tilt of a house. Figure 20(b) shows a house tilted and settled as a result of liquefaction. In the building supported by firm foundations such as piles, considerable vertical offset developed as typically displayed in Figure 22(c). The gap about 50cm between the building foundation and the subsided ground in its vicinity did result in breakage or tear-off of several pipelines and cables entering the buildings such as water and gas supply, sewage pipe, and electricity and telecommunication cables. The breakage of the lifelines as above was fatal for continued maintenance of operational conditions for the buildings. A conceptual picture illustrating the malfunctioning of sewage pipeline is given in Figure 23

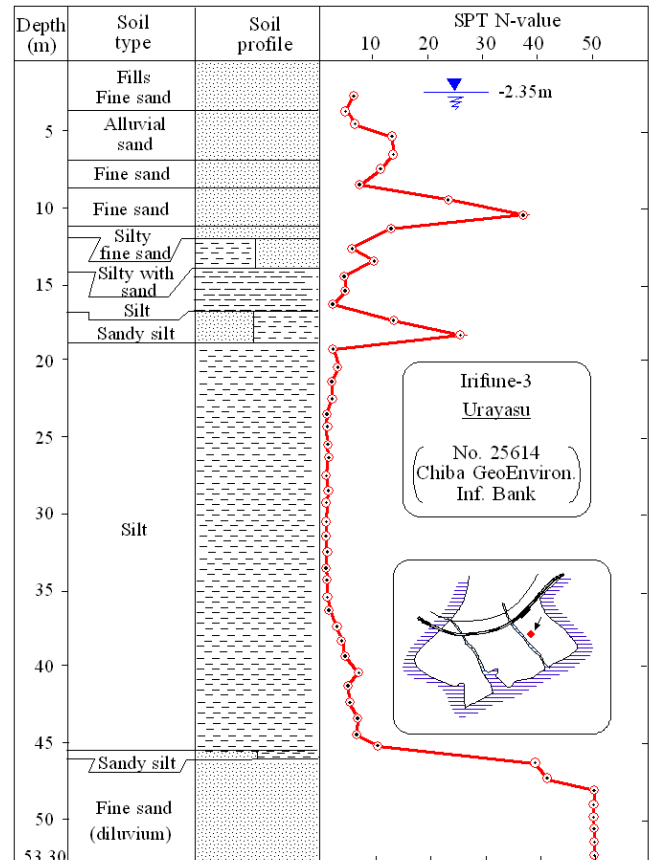


Figure 18 Soil profile at Irifune 3-chome

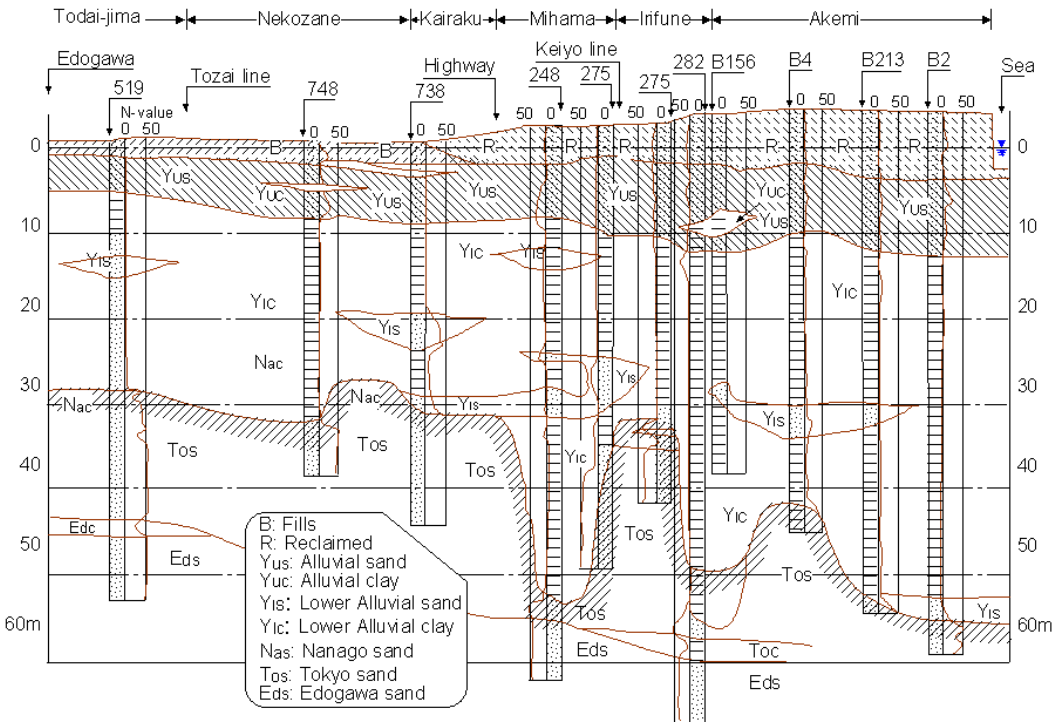


Figure 19 Cross section D-D' in Figure 20 (from the Urayasu city office)

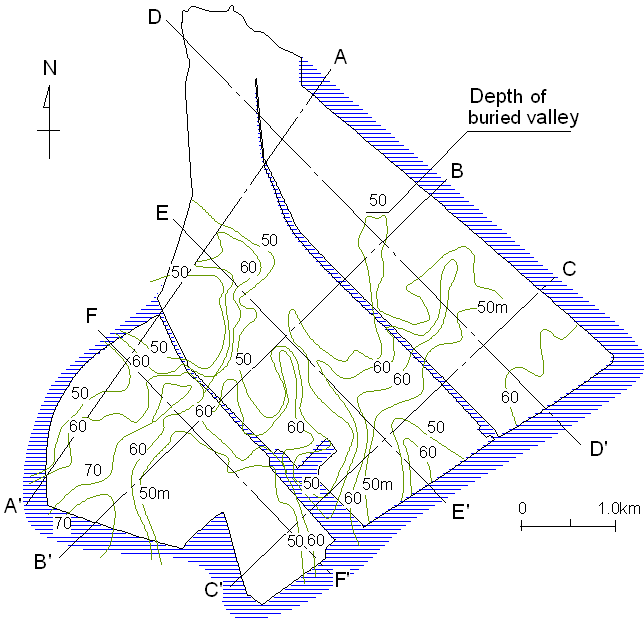


Figure 20 Counter lines of valleys buried by alluvial deposits at Urayasu (from the Urayasu city office)



(a) Ground devastation in a residential section



(b) Floating of sewage manhole



(c) Spreading of sand over the road



(b) Sinking of an apartment house

Figure 21 Destruction of the ground due to liquefaction
(Offered by Urayasu city office)



(a) Settlement and tilt of a building.



(c) Offset between an undamaged building and settlement of the surrounding ground

Figure 22 Settlements and tilt of houses due to liquefaction
(Offered by Urayasu city office)

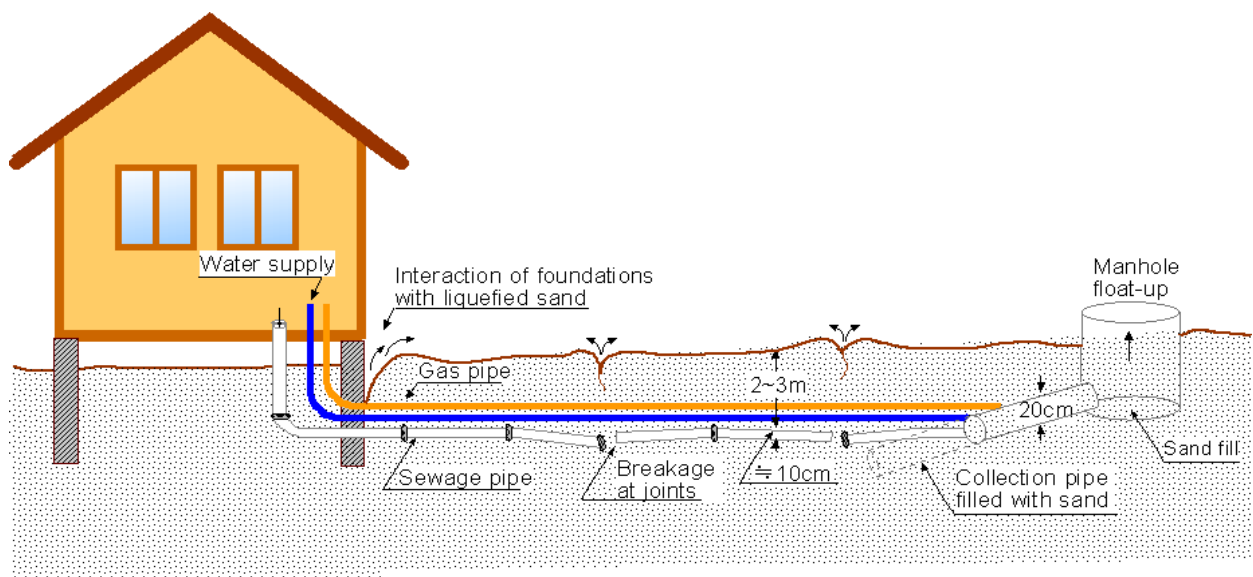


Figure 23 Conceptual picture illustrating the damage of lifelines entering houses

5.2 Flow slides in waterfront area

There were some places in Urayasu City where large lateral displacements took place in the man-made fills behind the seawalls. Most spectacular damage of this type did take place in a gentle seaside slope behind the cemetery park located in the east-south edge of Hinode-8 (see Figure 15). Views of the damage in this slope are demonstrated in the photos of Figure 24 where it can be seen that a tremendous amount of silt-laden sands had ejected from several lines of fissures developed in parallel to the coastal line. A cross section in the middle of the slide area is roughly depicted as shown in Figure 25 where it is noted that there is a concrete-made seawall which is supported by rows of sheet piles. In front of the wall, there is a paved apron about 30m wide which is retained by the reinforced concrete wall and wave-breaking concrete blocks stacked on the seaside.

It appears likely that the man-made fills underneath the apron as well as those in the landside of the concrete seawall were all placed while lightly compacting sandy soils by means of bulldozers which remained still loose enough to trigger liquefaction. Coming back to the landslide in the cemetery park shown in Figure 25, one can envisage that the overall liquefaction through the distance of 110m in the cross section is accompanied by the lateral flow of the liquefied silt-laden sand.

In fact, the seaward edge of the asphalt-paved apron was seen having moved laterally through 1.0 to 1.5m, thereby breaking off its edge and falling down into the wave-breaking concrete blocks. As viewed in a photo in Figure 24(b), traces of sand ejection were observed through open seams of pavements in the apron.

The concrete seawall in the landside was also seen having deformed seawards by 20~50m as indicated in Figure 25, which is indicative of the mass of liquefied sand deposit having slid along the likely sliding plane as indicated by a dashed line in figure.

5.3 Ichikawa City

The city is located immediately east of Urayasu city as shown in Figure 14. The major damage was induced by liquefaction in the area of man-made coastal zones in the south as indicated in Figure 27. Shown in the photo of Figure 26(a) are sand boils and resulting ground settlements observed in the belt zone about 3m wide just behind the sea protection wall along Shiohama. The robust seawall unit consists of stacks of stones (about 1.0~1.5m in size) in front of the sheet-pile supported concrete wall. Although there are some variations in cross section, the coastal line along the Tokyo Bay in this area had been firmly protected by this type of compound structure and, hence performed well without damage.



(a) A depression of the ground inland of the seawall - a view to the East



(b) Sand gushing from a seam of pavement in the apron in front of the seawall – a view to the west



(c) Damage at the edge of the apron - a view to the west

Figure 24 Lateral spreading towards the sea behind the cemetery park at Hinode-8, Urayasu

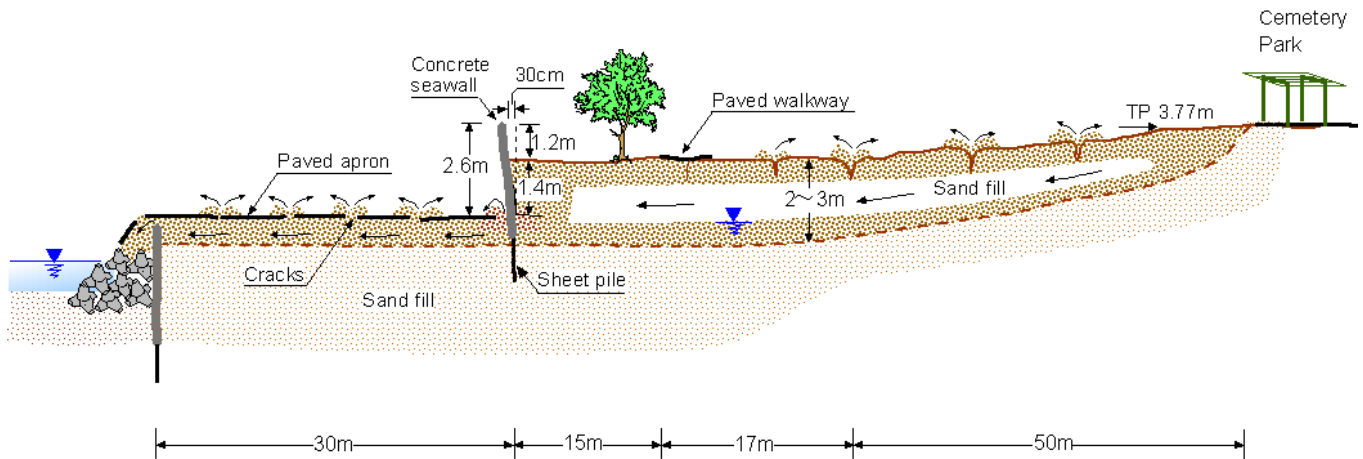


Figure 25 A conceptual picture of the lateral spreading at the seaside zone in the south of the Cemetery Park, Urayasu



(a) A view of the undamaged seawall (centre) with extensive liquefaction at inland walkway - a view to the east



(b) Damaged steps in the fishery port with the lateral displacement of about 50cm - a view to the east.

Figure 26 Ground damage at Shiohama, Ichikawa, Chiba-City

Although there was strong shaking of the order of 150~200gal plus the following rise of about 1.0m in the sea level due to Tsunami, the seashore protection structures as above remain almost intact and fulfilled its intended purpose.

Shown in the photo of Figure 26(b) is the damage to fishery port in Ichikawa involving lateral displacement of the order of 30~50cm at the crest level. Considerable evidences of liquefaction were observed along the belt zone along the seashore road as shown in the map of Figure 27.

5.4 Funabashi City

Most of the shoreline structures are warehouses, industrial buildings, quaywalls of private industries

which were difficult to make access for site investigation, but these appear to be protected by the pile or H-beam supported concrete seawalls. Most pronounced damage was to houses and facilities in Funabashi Seashore Park located behind the sand dune of the beach line as shown in Figure 27. Overall ground subsidence of the order of 50cm occurred due to extensive liquefaction. Outdoor small shopping houses supposedly supported by flat foundations sunk by 50cm due to inundation of sand as shown in the photo of Figure 28(a).

Along the beach line at the Park, cracks or fissures of the ground were observed, as displayed in Figure 28(b), resulting from lateral flow of the liquefied sand.

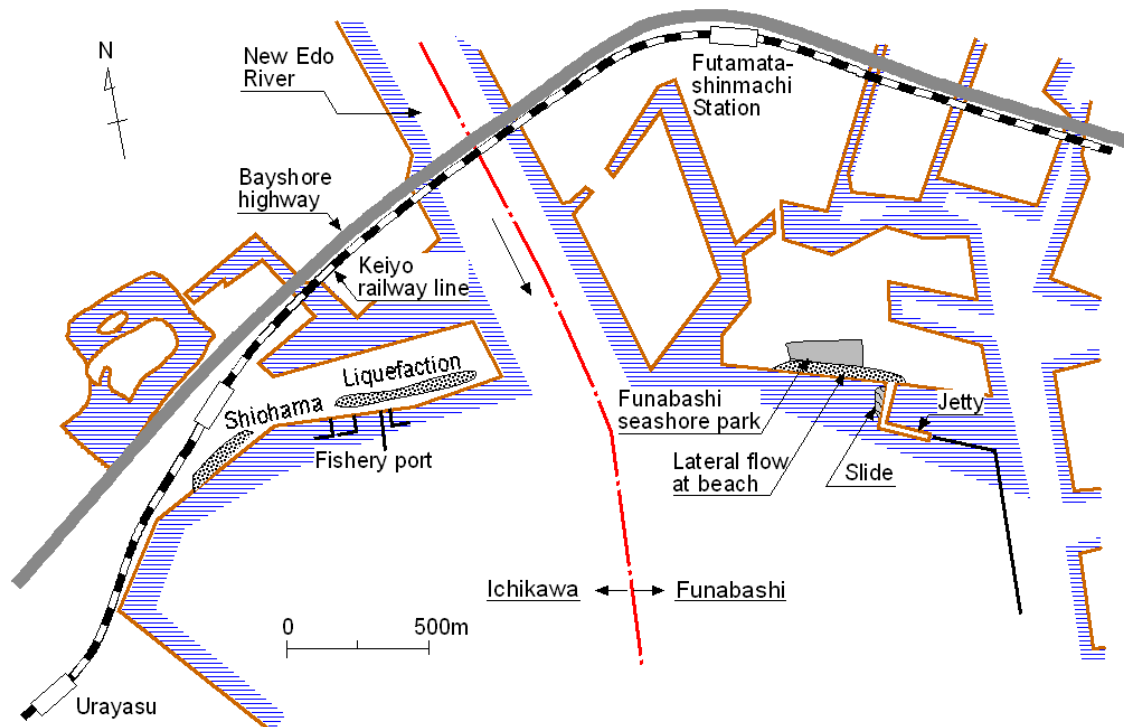


Figure 27 Ichikawa and Funabashi City



(a) Sinking of a small house with overall spreading of liquefied sand - a view to the sea



(b) Lateral spreading of the beach in front of the Seashore Park - a view to the west

Figure 28 Damage to the ground by liquefaction in the Seashore Park at Funabashi

5.5 Makuhari City

The city of Makuhari had been developed in 1970's over reclaimed lands, based on well-conceived city planning and all the buildings, parks, infrastructures etc. are arranged in an orderly manner. These facilities themselves were constructed on stabilized soils and supported, in addition, by different types of piles embedded to stiff strata, and did not experience any damage. However the lands such as roads, squares,

parks, schools, play grounds and parking places had not been improved and did experience extensive damage due to the ground liquefaction involving cracking, sand ejection and overall settlements of the order of 30 to 50cm. Particularly noticeable was the vertical offsets between the periphery of pile-supported buildings and the paved aprons or sideways in their vicinity.

There is a river about 20m wide called Hanami River south of Makuhari. On Makuhari side near the mouth of the river, the area including the road about 100m

wide and 200m long, as indicated on the map of Figure 27, suffered a subsidence of about 50~100cm due to liquefaction. The photo in Figure 28(a) shows a vertical offset of about 50cm that had developed in the park through a distance of about 200m. The photo (b) showed the vertical offset in front of the 10 storey building on the north side of the road. It may be seen that the vertical gap of the order of 50~70cm in front of the building produced damage to facilities such as portal steps and fences. In the Traffic Park, east of the above mentioned area, traces of considerable liquefaction was observed on the ground surface, including development of widely open cracks and sand boiling. The photo in Figure 31(a), viewed from west shows open cracks, as wide as 0.7~1.0m, running through the park in parallel to the line of the Hanami River. Shown in Figure 31(b) is the end of the cracks, viewed from north, blocked by a one-storey office building. It is of interest to note the large push-in of the ground having developed along the periphery of the building due probably to punching interaction between the concrete foundation and the surrounding soil deposits that had liquefied. The depth of the push-in was 50~70cm relative to the flat level of the surrounding ground.

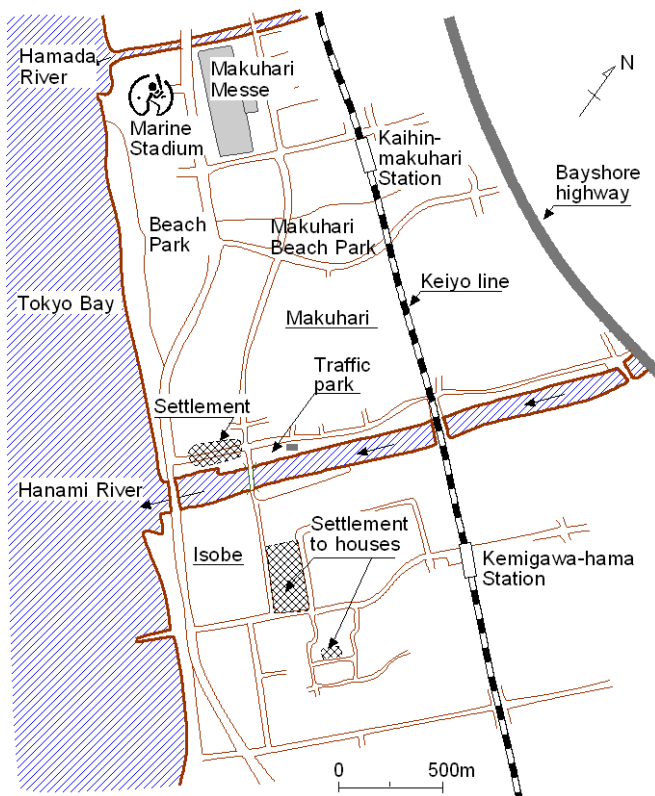


Figure 29 Makuhari-Isobe districts, Mihama-ku, Chiba

The Traffic Park is bordered by the line of bush fence on the walkway about 1.0m lower in elevation which is located along the Hanami River immediately on the south. The concrete wall along the river was turned down as viewed in the photo of Figure 32(a). There was a considerable amount of sand that had spread over the apron one step down the walkway as shown in Figure 32(b). It is highly likely that the liquefied sand over the park premise about 2m higher in elevation had flowed underground towards the Hanami River and pushed out the walkway wall, resulting in the collapse of the wall and spreading of the sand over the apron. The scenario as such is displayed in the cross sectional sketch shown in Figure 33.



(a) Vertical offset in the park on the south side of the road



(b) Small facilities at the portal of the building

Figure 30 Damage features associated with large settlements over the area at the north bank of the Hanami River near its mouth



(a) Open cracks through the Traffic Park - a view to the east



(a) Collapse of the wall north of the Hanami River – a view from the west



(b) Punching settlement along the periphery of the office building in the Traffic Park - a view to the south



(b) Collapse of the wall north of the Hanami River – a view from the east to the sea

Figure 31 Open cracks through the premise Traffic Park and the punching settlement of the office building

Figure 32 Failure of the walkway wall on the north of the Hanami River

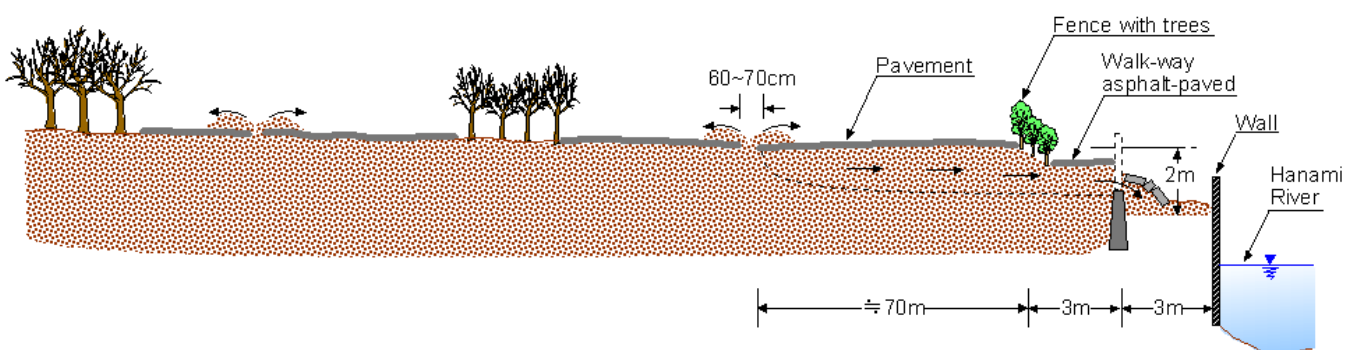


Figure 33 A conceptual sketch across the Traffic Park for the lateral flow of the liquefied sand towards the Hanami River



Figure 34 Annular sag in the liquefied ground surrounding the column for the cross-road walkway bridge

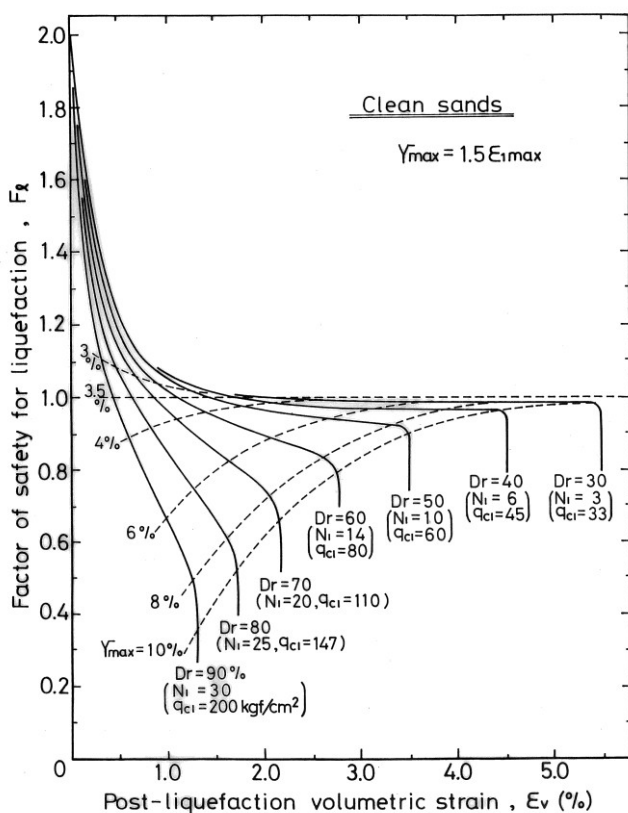


Figure 35 Chart for evaluating the volumetric strain as functions of the computed factor of safety, F_{ℓ} , and the density of sand in terms of SPT N-value (Ishihara-Yoshimine, 1992)

In Makuhari district there are a number of high-rise buildings and modern infrastructures which had been

constructed and arranged based on well-conceived grand design of the city. Since the area was reclaimed over the period of 1960~1980 by sands mainly obtained by dredging the bottom of shallow-depth nearshore waters, the soil profiles consist of loose sands with SPT N-value of the order of 5 to depths of 5~7m, underlain by soft silty clay layers of alluvial or Holocene origin having a thickness of about 10~40m. All of these facilities had been built up on the foundations supported by various types of piles embedded to the stiff deposits about 20~40m deep.

In addition, the near-surface sand deposits had generally been compacted by various methods in order to provide sufficiently competent resistance of the foundation pile system against the lateral force induced by strong earthquakes. In contrast, a large portion of the reclaimed lands used for roads, parks, playgrounds, tree-planted sideways, green belts etc. had been left unstabilized.

Thus, while there was practically no damage to the well-engineered structures, extensive damage developed due to liquefaction over the wide-spread areas which had been left intact without implementing stabilization of sandy deposit. The overall settlements of the order of 30 to 50cm were observed here and there with cracking or offsets in the pavements and corner stones or sometimes with wavy distortion of road alignments. These injuries were always accompanied by sand spurting or boiling, indicating apparently that the occurrence of liquefaction was the major cause of the distress. Shown in Figure 34 is a photo showing annular-shaped sag of the liquefied sand surrounding a 1.5m in-diameter column supporting the pathway bridge pier over the road near the entrance to the parking area in Marina Baseball stadium behind the coastal line (see Figure 29).

6. SETTLEMENT ANALYSES BY THE SIMPLE METHOD

In order to provide a first-hand interpretation for the occurrence of liquefaction in the area of Tokyo Bay, estimate of the factor of safety and the settlement of the ground surface were made by utilizing the chart correlating the volumetric strains with the factor of safety against liquefaction, F_{ℓ} , and SPT N-value. The chart used for the estimate is shown in Figure 35. In performing the analyses, several assumptions were made as follows.

(1) Estimate was made of the cyclic shear strength defined as the cyclic shear stress causing 5% double-amplitude shear stress, using the formulae stipulated in the Japanese design code for the bridge foundation.

(2) The peak acceleration of 0.16g was chosen for evaluating the factor of safety, which is the value recorded at the K-Net station in Urayasu as indicated in Figure 7.

(3) When converting irregular time histories of recorded accelerations to the constant-amplitude cyclic strength, the coefficient $C_2 = 0.6 \sim 0.7$ has been applied to the peak acceleration to obtain the cyclic strength corresponding to 20 cycles of uniform load application. This assumption was based on the duration of shaking of about 10~20 seconds which had been the case in the majority of large earthquakes encountered hitherto. In the case of the Great East Japan Earthquake, the shaking continued as long as 120~180 seconds. In such a long-duration of motion, the coefficient C_2 should be increased probably to a value of 0.8 to 1.0. Without any results of laboratory tests, at the moment, substantiating choice of appropriate value, the coefficient will be taken as being $C_2 = 0.8$ in the present analysis.

(4) There are many data on soil profiles in Urayasu city area which are publicized by the Chiba Geoenvironmental Information Bank. These are the data that have been obtained at various occasions in the period of 1960~1990 in the progress of the reclamation works. Thus, the depth of the ground water table at present may be different from that at the time of the borings. However, there is no way, unfortunately, to correct this change, if any. Thus, analysis of liquefaction was made herein by assuming that the ground water table remained unchanged since the time of boring.

(5) The liquefied sand in Tokyo Bay area contain 10~50% fines. Thus, correction had to be made for the SPT N-values. This was done by using the chart stipulated in the Japanese Bridge Foundation Design Code.

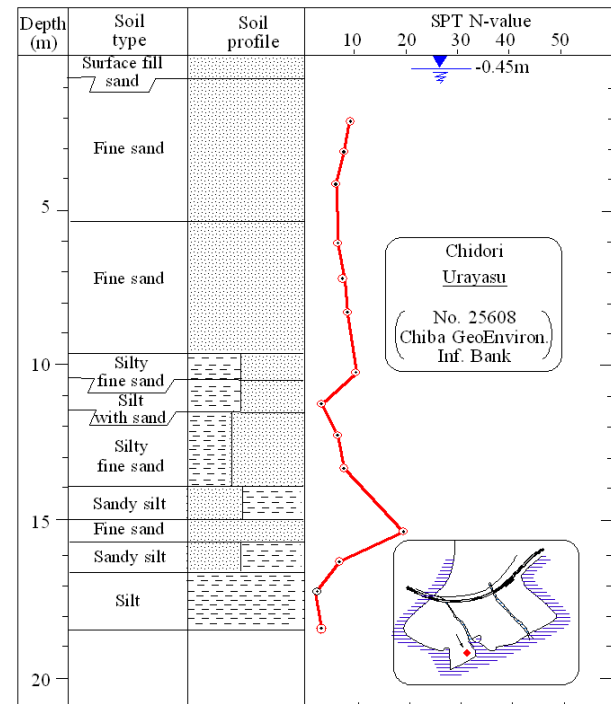


Figure 36 Soil profile at Chidori, Urayasu

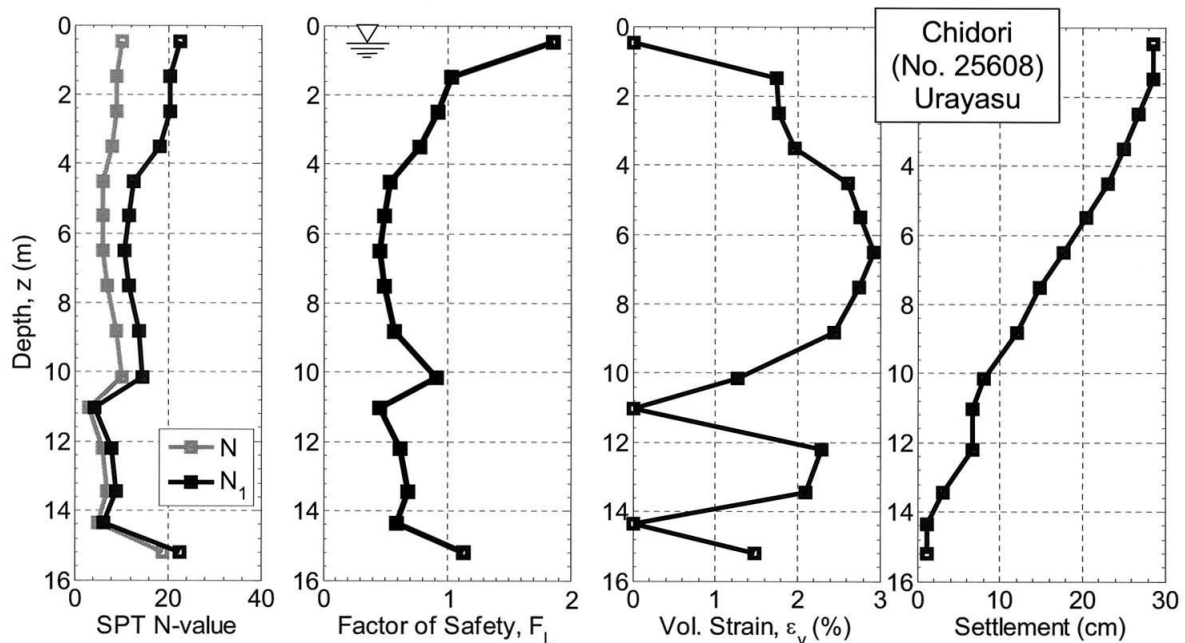


Figure 37 Analysis of liquefaction for a site at Chidori, Urayasu

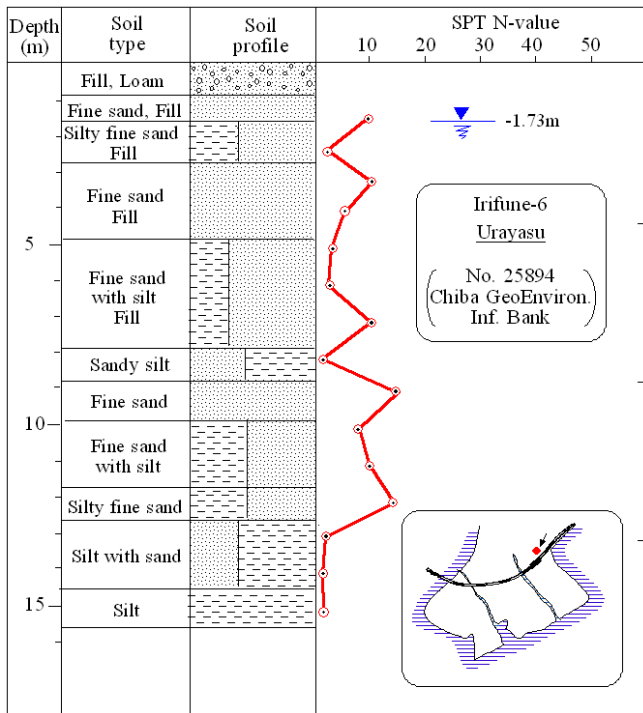


Figure 38 Soil profile at Irifune 6-chome, Urayasu

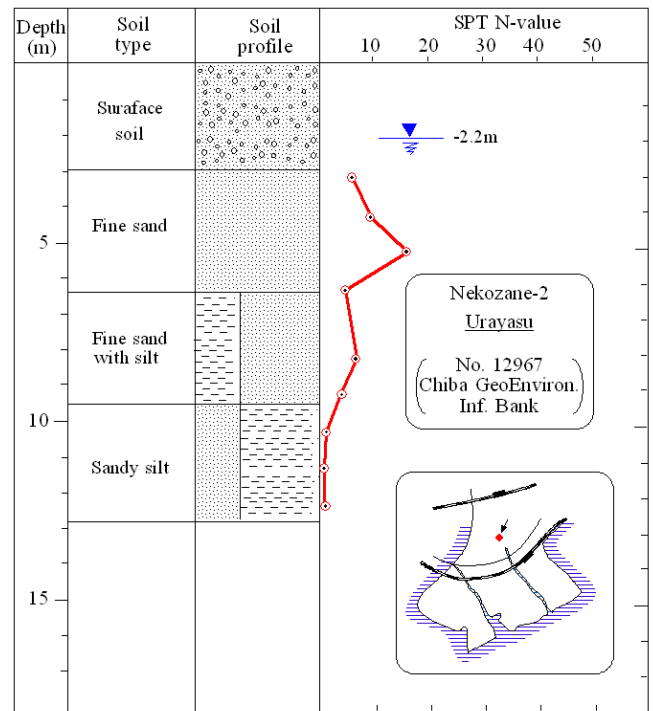


Figure 40 Soil profile at Nekozane 2-chome, Urayasu

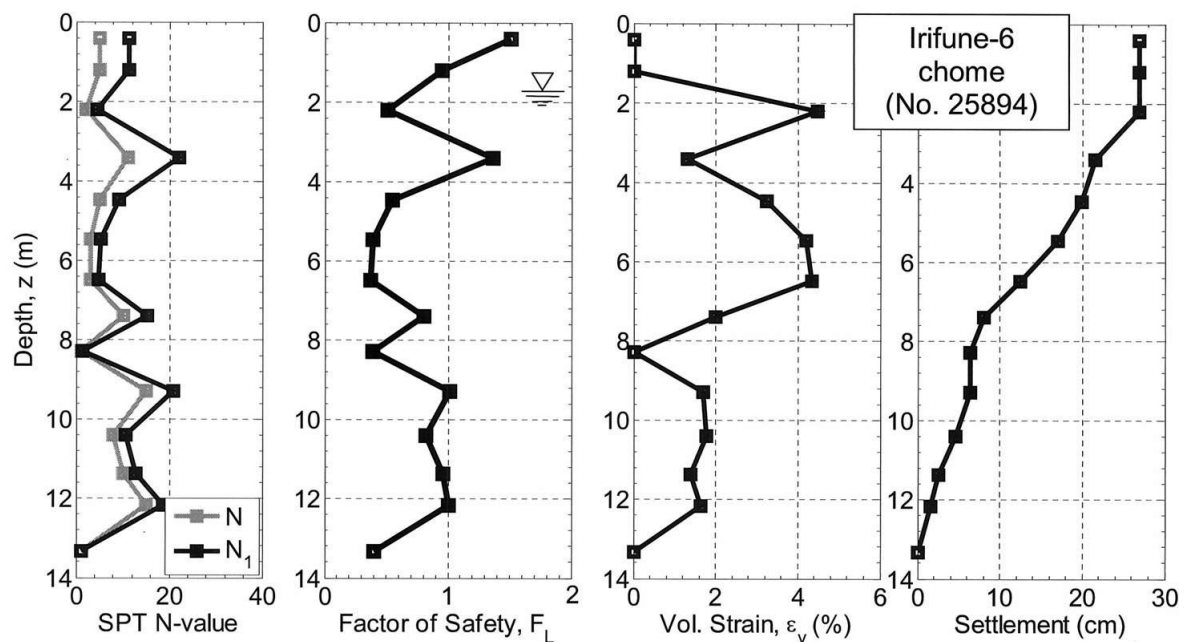


Figure 39 Analysis of liquefaction for a site at Irifune 6-chome, Urayasu

6.1 Settlements at sites of liquefaction in Urayasu

A site at Chidori was chosen as a place for liquefaction. The soil profile and its location are shown in Figure 36. The results of the liquefaction analysis are shown in Figure 37. The analysis was made for the near-surface sandy deposits to a depth of 15m. This is the site where signs of liquefaction were

visible at the time of the earthquake. The factor of safety against liquefaction is shown in Figure 37 to be less than unity generating the volume strain of $\varepsilon_v = 1-3\%$ through the depth of 2 to 11m. The settlement of the ground surface was obtained by integrating the volumetric strain at each depth, coming up with a value of 30cm.

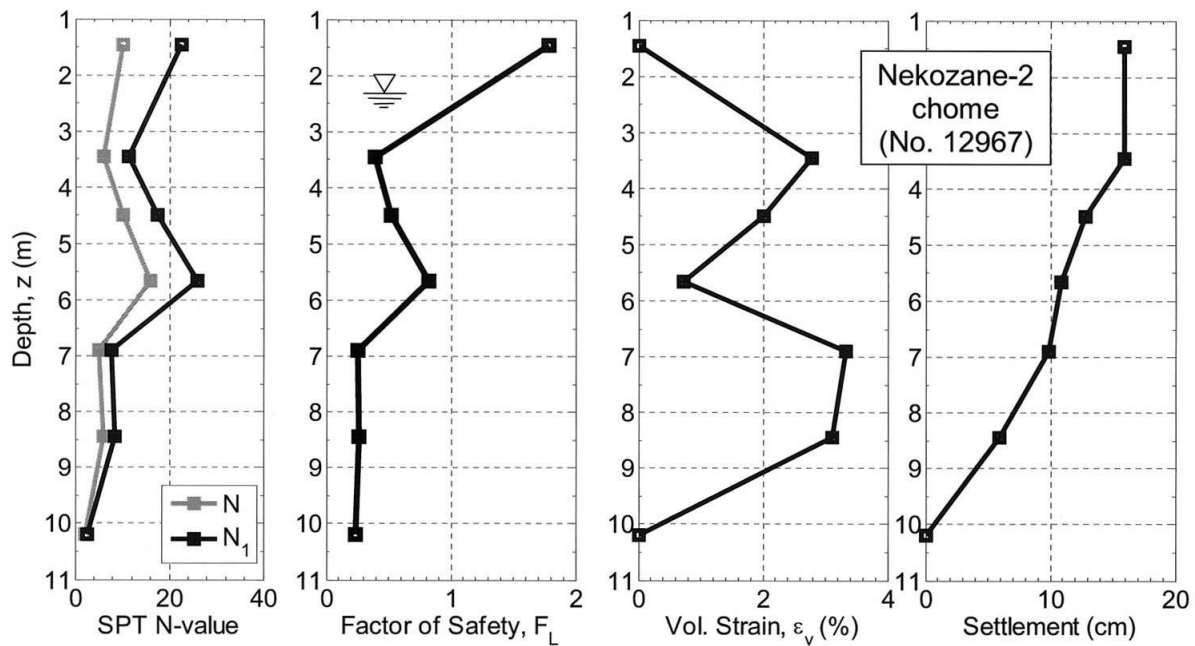


Figure 41 Analysis of liquefaction for a site at Nekozone-2chome, Urayasu

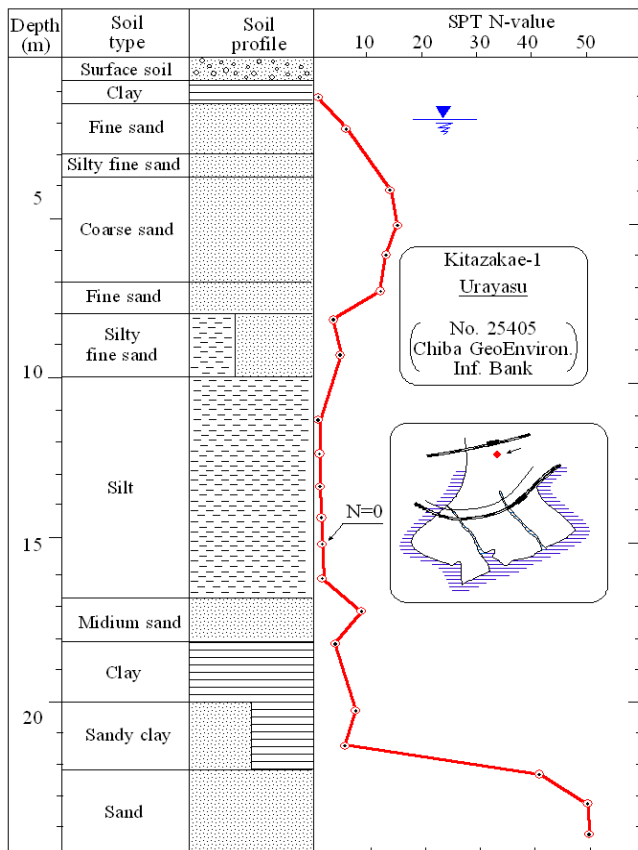


Figure 42 Soil profile at Kitazakae 1-chome, Urayasu

The actual settlement exactly at this place was estimated to be 50~70cm judging from the post-quake ground elevation relative to a nearby factory building

supposedly supported by piles. The value of the estimated settlements is displayed in Figure 44 versus the range of the settlement judged from observation after the quake.

Another example of comparison between the estimated and observed value was made for the site of Irifune 6-chome of which the soil profile is shown in Figure 38 together with the location in the inset. Results of analysis are shown in Figure 39. The actual settlement was inferred to be 30cm by comparing the post-earthquake ground level to the trace of the pre-earthquake ground surface marked on a pile-supported column of a nearby overpass walkway. As shown in Figure 39 the estimated settlement was 29cm indicating good coincidence with the actual value.

6.2 Settlements at sites of no-liquefaction in Urayasu

In the old city section of north Urayasu, no visible sign of liquefaction was observed at the time of the 2011 earthquake. The settlement analyses were performed for two sites in this area. One of the sites in Nekozone 2-chome has a soil profile as shown in Figure 40.

The results of liquefaction analyses are shown in Figure 41 where the settlement is calculated to be 16cm. This value is also plotted in Figure 43. Similarly analysis was performed for another site of no-liquefaction, viz., at Kitazakae 1-chome, where the soil profile consists of relatively dense sand of alluvial origin as indicated in Figure 42. The calculated settlement is shown in Figure 43 being of the order of 10~20cm.

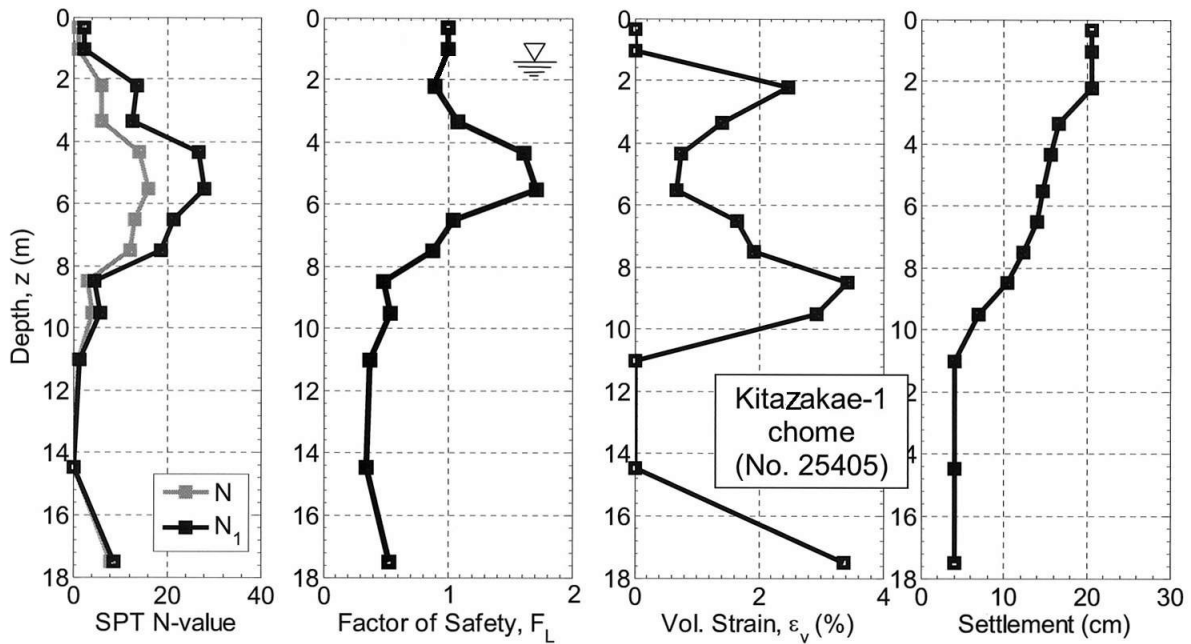


Figure 43 Analysis of liquefaction for a site at Kitazakae 1-chome, Urayasu

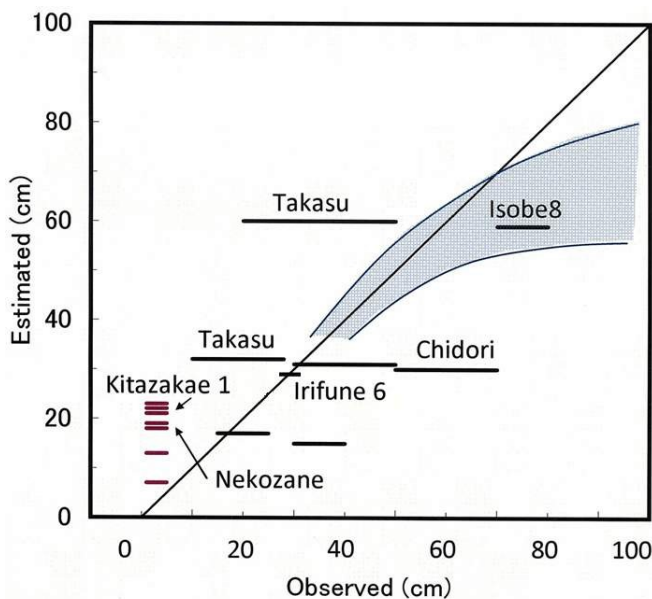


Figure 44 Comparison of estimated and observed settlements in Urayasu

6.3 Comparison of settlements between the calculated and observed values

There are several other sites in the city of Urayasu for which estimate was made by using the simple method as described above. All the data obtained from calculation are plotted in Figure 44 versus actually observed values. As there are several sites where actual values of settlements are difficult to determine uniquely, they are demonstrated in the form of ranges

of variation in Figure 44. Looking over the plots in Figure 44, one can recognize main points as follows.

(i) For the sites with no evident sign of liquefaction, the computed values of settlement are larger than those observed actually after the earthquake. There might have been some settlements which was not discernable, but the SPT N-values in the soil profile used for assessment of the volumetric strain may not reasonably reflect the ground conditions particularly consisting of alluvial sandy deposits where cementation or aging effects are predominant.

(ii) For the sites with apparent sign of liquefaction, the estimated settlements show values of 20~30cm which is coincident, by and large, with the actual values observed at the time of the earthquake.

(iii) In the range of larger settlements roughly in excess of 40cm, the observed settlements appear to show values which are greater than those estimated by calculation as indicated by shaded zone in Figure 44. Most likely, this is due to loss of the large amount of liquefied sand which ejected on the ground surface. In the current methodology, it would be difficult to take into account the amount of sands which ejects out of the ground when liquefaction occurs.

7. CONCLUSIVE REMARKS

There were several characteristics in the occurrence of liquefaction and consequence damage which are different from a number of cases experienced in the past earthquakes. These are summed up as follows.

1. It was probably the first case to the knowledge of today's geotechnical engineers to observe such a widespread occurrence of liquefaction in the area more than 350km distant from the epicentre. This may be attributed to the rarely encountered huge magnitude-scale earthquake as great as $M=9.0$.

2. Corresponding to the large scale event, the duration of shaking was as long as 2~3minutes. In the light of the 10~20 seconds duration hitherto encountered in the magnitude 7~8 events, the shaking duration in the East-Japan Great Earthquake can be cited as extraordinarily long. This fact appears to have made the damage level worst, as compared to any other cases of liquefaction ever encountered.

3. While there was severe damage by liquefaction in the newly reclaimed area, practically no sign of liquefaction was observed in the old part of Urayasu city where soil deposits consist of sandy soils with alluvial origin. It is likely that for clean sand, effect of aging may not be so pronounced. However, fines such as silt or clay tend to develop cementation more readily. Therefore, in old deposits comprised of the alluvium, it is conceivable that the effects of aging tend to act for solidifying deposits more pronouncedly for portion of fines contained in the sands. This could perhaps be a reason why there was a sharp contrast regarding the liquefaction between areas of alluvial deposits and newly reclaimed sections in Urayasu city.

4. It should be noticed that the effects of aging on fines may not be precisely reflected in the penetration resistance such as SPT or CPT. There should be some other means or parameters to quantify the aging effects. This point would be a new aspect of the problem to be pursued in future.

5. When newly deposited sand contain fines, the fines-laden liquefied sands tend to flow easily permitting a large amount of sands to be ejected on the ground and hence, to deposit as thick as 50cm on the ground surface. This fact implies, on the other hands, that several small cavities or holes were left hidden underneath the surface soils. This will create big problems in future for the long-term maintenance of infrastructures under the ground.

8. ACKNOULEGEMENTS

In preparing this paper, the information on the progress of reclamation in Urayasu city and general features of the damage was given by the courtesy of Mr. Ichiro Ishii of the Urayasu city office. The manuscript was arranged by Mrs. K. Ino of Kiso-Jiban consultant. The authors wish to express their deep thanks to the office and persons as above.

9. REFERENCES

- Chiba Geo-Environmental Data Bank,
<http://www.pref.chiba.lg.jp/pbgeogis/servlet/infobank.index>.
- Hayes, G. P., Earle, P. S., Benz, H. M., Wald, D. J., Briggs, R. W., the USGS/NEIC Earthquake Response Team, (2011), "88 Hours: The U.S. Geological Survey National Earthquake Information Center Response to the 11 March 2011 Mw 9.0 Tohoku Earthquake", *Seismological Research Letters*, Vol. 82, pp. 481-493.
- Ishihara, K. and Yoshimine, M. (1992), "Evaluation of Settlements in Sand Deposits Following Liquefaction during Earthquakes," *Journals of Japanese Geotechnical Society, Soils and Foundations*, Vol. 32, pp. 173-188.
- Towhata, I., Goto, H., Kazama, M., Kiyota, T., Nakamura, S., Wakamatsu, K., Wakai, A., Yasuda, S. and Yoshida, N. (2011), "On Gigantic Tohoku Pacific Earthquake in Japan," *Earthquake News, ISSMGE Bulletin*, Volume 5 Issue 2.
- Urayasu City Official Site
<http://www.city.urayasu.chiba.jp>
- Yasuda, S. (2011), "Liquefaction-Induced Damage in Kanto Region," *Report of Investigation for the Tohoku Off-Pacific Ocean Earthquake*. Japanese Geotechnical Society (in Japanese).